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The Application of Roller Compacted Concrete to Pavements for Heavy Truck Routes in Ontario

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The Application of Roller Compacted Concrete to Pavements for Heavy Truck Routes in Ontario

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Abstract:	This report contains a brief state-of-the-art on roller compacted concrete (RCC). It is not slipformed as conventional concrete pavement, but compacted by rollers similar to the construction of bituminous pavements. RCC has a low moisture content and a low water/cement ratio, beneficial for the strength of the concrete. The findings of an experimental programme on durability and fatigue properties of RCC are presented. In these laboratory tests it was found that there is a larger than usual scattering of fatigue strength which has to be taken into account when determining design thickness. In summary, RCC can compete with conventional concrete; however, the cost savings are much less than usually suggested in the literature due to the fact that RCC pavements must be 10 to 20% thicker than conventional ones.
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1/ Introduction

1.1/ General

Roller compacted concrete (RCC) refers to the technique using a zero slump Portland cement concrete, placed by common asphalt paving equipment, and compacted by a vibratory roller to form a high strength concrete. Although the technology for RCC has been available since the early 1940s [1], its identification with cement-treated bases (CTB) and soil cement had suppressed the development of this technology. It was through successful applications of RCC on dam construction that interest was eventually generated in the pavement construction industry. Initially, RCC was looked upon as a technology for constructing strong, but relatively inexpensive pavements (e.g., dry-log sorting and intermodal yard pavements) using marginal or low grade aggregates. However, the more recent literature suggests that RCC is evolving to become a technology with tighter controls on aggregate quality and gradation, and on mix design and construction than was originally intended for this type of construction.

1.2/ Background

The development of concrete pavement practice for Ontario highways has undergone several stages which are well documented in the Minitry of Transportation (MTO) literature (see, for example, References 2 and 3). The first concrete sections were laid by hand with no reinforcement, on little or no base course and with contraction joints placed every 5 to 7 m to control cracking. The inadequacies of the earlier designs became apparent as traffic levels and loading increased during the 1950s; that is, pumping of subgrade led to a loss of fine material by erosion, a loss of subgrade support, and subsequent pavement failure. An increased use of de-icing chemicals led to serious salt scaling. Subsequent corrective measures over the years resulted in the use of air-entrained concrete and high quality aggregates to improve durability, and in the introduction of subbase courses, load transfer devices at joints and reinforcement to reduce pumping, improve load transfer across joints and increase overall load carrying capacity of the concrete pavements.

Prior to 1968, heavy-duty concrete pavements were 225 mm (9 inch) thick reinforced slabs on a 225 mm subbase. However, owing to the increased incidence of transverse cracks, and spalling and faulting at joints and cracks, the Ministry design for such pavements was altered to a 225 mm (9 inch) thick plain concrete slab on a 127 mm stabilized base. This practice was discontinued after 1971 because of the high cost of construction. As a result, no new Portland cement pavements

were constructed until 1982 [3] when it was decided to investigate the feasibility of implementing newer concrete design and construction developments in the industry.

Over the past decade, the trends in the transportation industry to higher wheel loads and newer 'super single' tires which improve fuel efficiency have contributed to rutting of flexible pavements on heavy truck routes, causing increasing concern for both user safety and highway serviceability. The severe pavement rutting as observed on the eastbound Highway 401 between Oshawa and Trenton, although partly due to inadequate asphaltic concrete mix design, has been attributed to both the increase in volume of truck traffic and changes in pavement loading characteristics. A recent Ministry survey has found that 81% of trucks are outfitted with the newer technology radial ply tires which have on average 40% greater tire pressures when compared with the older bias-ply tires. With the recent development of the 'super-single' truck tire, it is anticipated that the rutting problem will intensify, thereby substantially reducing the life of a pavement structure [4].

As a result of the concern for the problems associated with the rutting of flexible pavements and the improved cost-effectiveness reported of newer rigid pavement designs, MTO has renewed their interest, at least in part, in the use of Portland cement concrete for paving applications. In 1982 a test section was constructed on Highway 3 to evaluate alternative Portland cement concrete pavement designs. More recently, the Ministry has decided to investigate the potential of using RCC as a cost-effective high quality component for composite pavements. This report describes the study which was completed for the MTO on roller compacted concrete.

1.3/ Objectives and Scope

The main objective of the study was to evaluate the potential of RCC composite pavements as a cost-effective means for reducing pavement rutting. Implicit in this objective was that any solution should be practical and able to be implemented by local contractors using locally available materials and construction equipment. The study was reduced to performing the following tasks:

a) Complete a literature review to assess the current state-of-the-art on RCC and to identify differences between RCC, conventional Portland cement concrete (PCC), and lean concrete base (LCB) pavements.

b) Carry out an experimental research programme to compare the strength and durability properties of conventional concrete (slump) and RCC (zero slump) using various Ontario aggregates.

c) Compare thickness design and material specifications for RCC and plain concrete pavements.

2/ Literature Review

2.1/ General

Roller compacted concrete has evolved rapidly during the past decade from a technology which was principally used for dam construction to one used for heavy-duty pavements (see, e.g., Reference 5). The development of RCC has resulted in a Portland cement concrete incorporating marginal, less expensive aggregates, but which exhibits strengths similar to those of conventional concrete utilizing more expensive, high quality mix components. The following sections provide a brief state-of-the-art, prepared in part by JEGEL (John Emery Geotechnical Engineering Ltd.), on RCC mix design, strength properties, placement techniques, and quality control. The term RCC is used both as a short form for the technology and for the material manufactured via this technology. Only some of the more relevant papers are referenced.

2.2/ Mix Design

The basic components of RCC are the same as those of conventional or lean concrete; i.e., binder consisting of Portland cement and mineral aggregate. Whereas the conventional concrete mix designs are based on achieving a certain slump for a given water/cement (w/c) ratio, which is an indicator of strength, RCC mix designs are most often based on maximizing compacted density by varying water content for a given cement content (see, e.g., Reference 6). Owing to the differences associated with the placement techniques and methodologies for determining the optimum constituent proportions, roller compacted concretes have much lower water contents; but, with less cement, have similar water/cement ratios when compared to conventional concretes, as shown in Figure 1. The optimum water content for RCC is generally established by using the Proctor-type method (ASTM D558 or D1557) which is used for soil cement. It should be noted that to achieve a suitable aggregate/cement (a/c) ratio, for a given water/cement (w/c) ratio and slump, water reducing additives are often required to achieve suitable conventional mix designs. It is generally assumed that additives are not required for RCC.

Although aggregates can be blended to give a desired gradation for RCC, most applications to date have made use of single aggregate mixtures, thereby reducing the cost of RCC. The recommended gradations for RCC are similar to those of Granular A (or B) as shown in Figure 2. In order to reduce the potential for segregation it has been found desirable to limit the maximum aggregate size

to that passing the 19 mm sieve and to specify 5 to 14% fines passing the 75 μm (#200) sieve size. The more recent literature suggests that aggregate durability should be the same as that used for conventional high quality concrete [7]. The addition of pozzolonic material such as flyash has been used for conventional concrete mixes as a workability enhancer and as partial replacement for the Portland cement itself. In the USA, several projects have made use of flyash as partial replacement to reduce the cost of RCC (see, e.g., Reference 1). However, research in Sweden by Andersson [8], suggests that flyash addition to RCC may have detrimental effects on the freeze-thaw durability of RCC. Instead, silica fume has been recommended as a replacement. It should be noted that the negative effect of the flyash on RCC may have been partially due to the properties of the Swedish flyash itself, which may be quite variable.

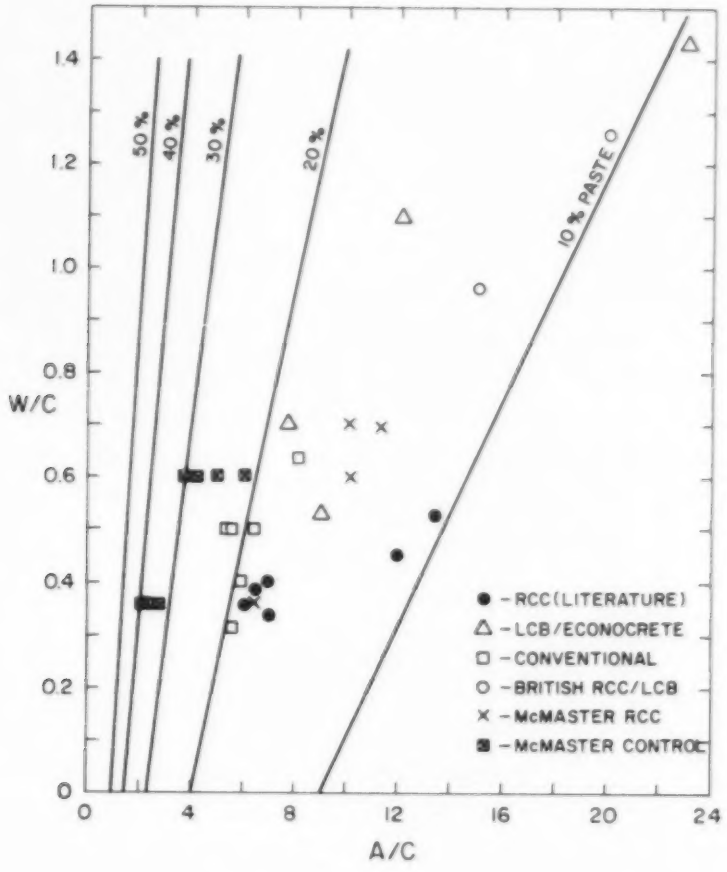


Figure 1/ Comparison of mix proportions for various concretes

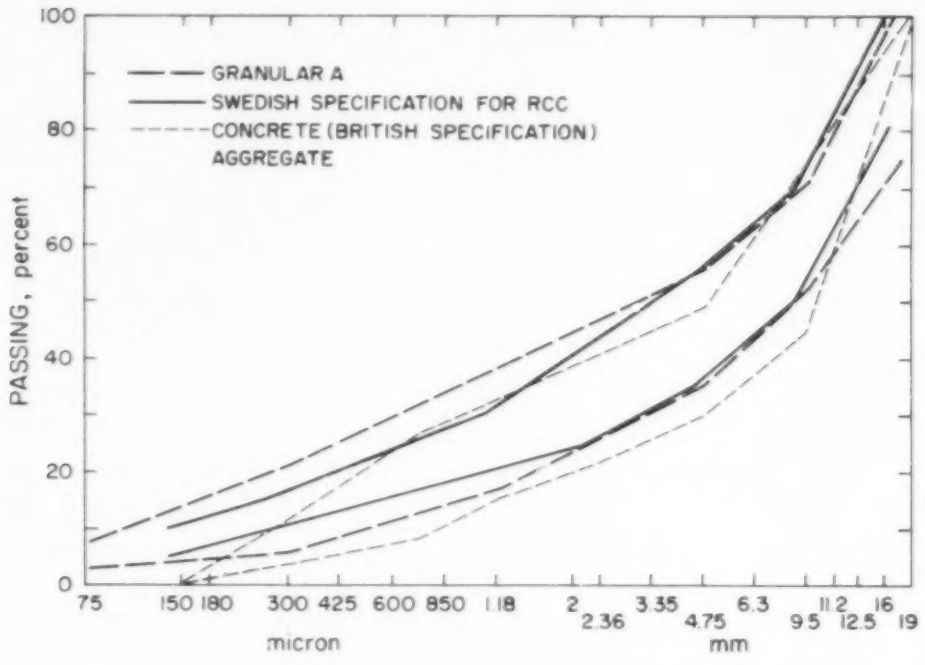


Figure 2/ Comparison of aggregate gradation bands for RCC, Granular A, and conventional concrete

In early projects, the cement content for RCC was selected empirically on the basis of the minimum considered to be a 'safe level' and experience with conventional concrete [9]. Other literature suggests that the cement content selection should be based on the flexural strength of laboratory cast beams. Flexural strengths after 28 days typically range from 4.1 to 4.8 MPa [10]. This range of strengths is less than the minimum flexural value of 4.8 MPa recommended for conventional concrete pavements which are designed in accordance with Ontario Provincial Standard Specifications OPSS 1350 (assuming that the achieved 10-day flexural strength is about 75% of the 28-day strength).

Durability of RCC mixes, which is an important component of this study, has generally received only secondary consideration for mix design. Laboratory testing carried out by the U.S. Army Corps of Engineers [11] and by the Swedish Cement and Concrete Research Institute [12], has

demonstrated that RCC can exhibit good frost resistance. Andersson [7,8] suggests that RCC may be considered frost resistant provided that the relative compaction of RCC in the field exceeds 97%. Limited studies on the addition of air-entraining agents to RCC mixes for improving freeze-thaw durability have been inconclusive. Ragan [11] found that the air void system which develops in RCC may provide some frost resistance.

A high quality RCC mix should contain approximately 14% Portland cement (300 kg) with 5-6% water. It has more cement (more than twice as much), makes use of higher quality aggregates and requires better quality control when compared with cement treated bases (CTB).

2.3/ Strength Properties

Test results for RCC pavements have shown that compressive and flexural strengths of RCC are similar to those of conventional concrete. No clear correlations have been established between *in situ* and laboratory compacted strengths nor between the various strength measures; e.g., between flexural and compressive strength. Early studies suggested that *in situ* strengths were higher due to better compaction. However, a more recent study by Tayabji and Okamoto [13] has shown that laboratory compacted strengths are not necessarily less than those obtained from testing of core or saw-cut samples. Table 1 gives a summary of some results from studies on strength properties.

As may be noted in Table 1, some scatter exists when comparing possible correlations between flexural and compressive strengths from the reported studies. The differences in correlation may be partially due to differences in mix designs and the varying degree of compaction between the tensile and compressive strength specimens within each study. For a McMaster study, both indirect tensile and compression test specimens were manufactured exactly the same way, which resulted in a surprisingly high correlation. The Portland Cement Association recommends using a K value of 9 [15] which is a little higher than that recommended for conventional concretes (see, e.g., References 16,17). Swedish experience [8,12] has indicated that a relationship exists between the degree of compaction and the strength of RCC, as illustrated in Figure 3. A similar relationship was identified between percent compaction and flexural strength.

Table 1/ Comparison of flexural and compressive RCC 28-day strengths

Reference	Compressive Strength f'_c		Modulus of Rupture M_r		$K=M_r/\sqrt{f'_c}$ based on psi
	(psi)	MPa	(psi)	MPa	
[14] (cut)	2700	18.6	620	4.3	11.93
	5660	39.0	785	5.4	10.43
	3940	27.2	710	4.9	11.31
	5880	40.5	755	5.2	9.85
	4170	28.8	755	5.2	11.69
[13] (cut)	3820	26.3	670	4.6	10.84
	3660	25.2	595	4.1	9.84
	4390	30.3	625	4.3	9.43
	2480	17.1	480	3.3	9.64
[13] (molded)	4440	30.6	705	4.9	10.58
	401(	27.6	600	4.1	9.47
	4820	33.2	490	3.4	7.06
	3700	25.5	490	3.4	8.03
[11]	4780	33.0	830	5.7	12.00
	6250	43.1	730	5.0	9.23
	5740	43.1	680	4.7	8.98
	6900	47.6	650	4.5	7.83

1000 psi =1 ksi = 6.895 MPa

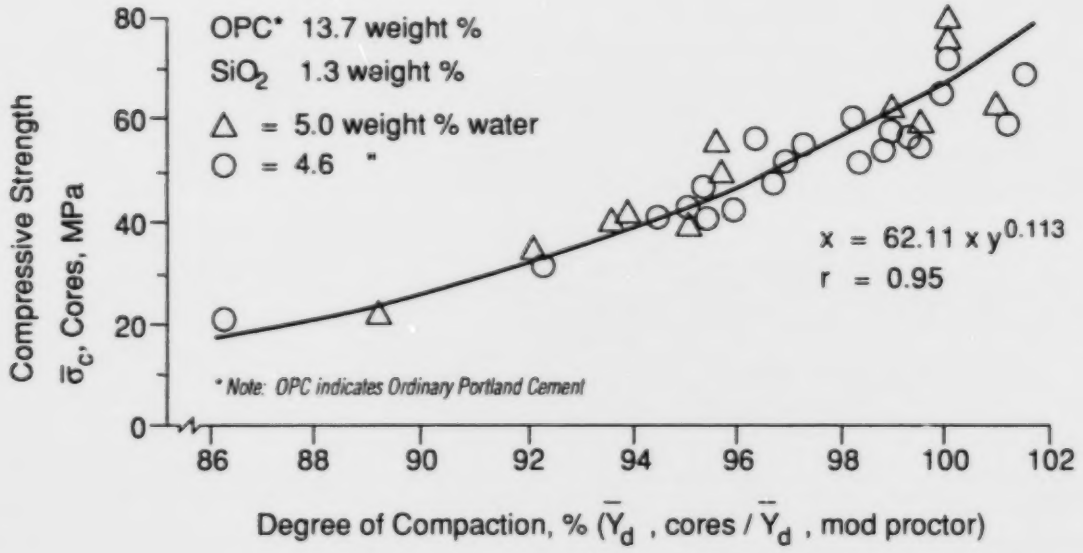


Figure 3/ Relationship between degree of compaction and compressive strength in core samples (tested at 28 days, containing crushed aggregate) (8)

Table 2 presents some typical mechanical properties for various concretes and untreated granular base material. It should be noted that the lean concrete mixes have much lower w/c than the other mixes. They are expected to have much lower strengths and stiffnesses. The properties of RCC and conventional concrete, as shown in Table 2, appear to be similar; however, results on fatigue strength of RCC suggest that it is less than that of conventional concrete [13].

Table 2/ Comparison of stress-strain properties, 28 days

Material	Unconfined Compressive Strength (MPa)	Modulus of Elasticity (MPa x 1000)
Roller compacted concrete	25-40	27-34
Conventional concrete	28-34	28-34
Lean concrete	5.2-15	14-21
Cement-Treated Base	3.4-5.5	6-18
Untreated Granular Base	*0-1.4	*0.08-0.25

* Function of confining pressure

2.4/ Construction and Quality Control

2.4.1/ Manufacture and Hauling

Roller compacted concrete is best mixed in a continuous-flow pugmill-type mixer used for manufacturing soil cement. It is recommended that a surge hopper be used to store and discharge the RCC into end-dump trucks. This allows the plant to maintain a uniform operational speed and, most importantly, reduces segregation of the RCC mix as it is transferred to the end-dump trucks. Owing to the coarser aggregates generally used and lower water content, care must be taken to minimize segregation while handling the RCC mix. Haulage and dumping of RCC should occur within 60 minutes of mixing [8]. Haulage trucks should be equipped with retractable covers to protect the material under extreme conditions, i.e., high air temperature and low humidity. Their purpose is to retain the already low moisture in the RCC mix. It should be noted that one should not add additional water directly to the mix prior to placement.

2.4.2/ Placement and Compaction

The laydown, finish and compaction operation time of RCC, which is similar to that of bituminous concrete, is approximately 30% of that required for proper construction of an equivalent conventional concrete pavement. With proper set-up and preparation time allowed for each lift and lane, the faster construction of RCC reduces the placement cost and hence, appears to make it more economical when compared to conventional Portland cement concrete placed using slipform pavers. A paver equipped with auger extension is capable of placing material up to 7.3 m (24 ft) wide and 225 mm (9 inches) thick without segregation of the RCC mix. However, without auger extensions, a paver should be restricted to the maximum width of the augers (typically 4.0 m).

(a) Dimensions

With thicker RCC pavements, say between 375 and 450 mm, two-layer or three-layer placement is recommended, with the upper layer being thinner than the other(s). This type of construction allows a consistent and sufficient degree of compaction to be achieved in the lower lift (approximately 98% of the specified density) with reasonable surface smoothness, and a closely controlled upper layer placement with good compaction and much better control on surface tolerance. Paving experience suggests that the second lift of RCC should be placed within 60 minutes of the first layer, thereby eliminating the need for any treatment of the first lift [1]. In comparison, conventional Portland cement concrete can be placed in single lifts of 460 mm thick using a slipform paver and a finisher for the surface preparation.

(b) Paving

Three types of pavers have been used to place RCC [18]. Before 1985, standard American asphalt paving machines were used. These machines imparted little compactive effort to the RCC; thus, vibratory rollers were then used to provide most of the required compaction. In 1985, a German firm started to market a heavy-duty paver capable of providing up to 95% modified Proctor compaction. The amount of rolling required was thereby greatly reduced. A middle-of-the-road American machine has since been introduced which provides up to 91% compaction. A recent study has suggested that the German machine may cause arching to occur in the compacted layer thus causing the top of the lift to be more dense than the bottom. It has also been suggested that with the newer American machine this problem may be avoided by allowing more compaction by the vibratory roller [18].

(c) Compaction

Compaction of RCC should be performed immediately after placement by the paver. Typically, a vibratory roller is used to compact the RCC to achieve the required overall density and then a pneumatic tire roller may be used to knead the surface to a dense state. Often, compaction is finished with a static pass to remove any roller marks left by the vibratory and pneumatic tire rollers. Experience has shown that a consistent, precise rolling pattern is required to ensure proper compaction and reasonable surface tolerance (see, e.g. Reference 8). Mix consolidation during compaction usually varies between 15 and 20%, depending on initial density which, in turn, depends on the paver used for the job. It is important that the subgrade be prepared in the same manner as for construction of a conventional base in order to ensure good compaction of the RCC. To reduce the contamination of RCC and to expedite construction, a 150 mm base is recommended. Compaction should be fully completed within two hours after mixing.

At the current stage of RCC development it is still unclear whether the lower layers, which should have started to set, undergo some damage due to the compaction of the upper lift(s). Based upon the successful *in situ* performance of RCC pavements, any such damage would appear to be minimal.

(d) Curing

Like conventional concrete, RCC must be moist-cured. This can be accomplished by several methods including the use of emulsions, membranes, wet burlap, water spray or wet sand [19]. Water spraying by a truck or a sprinkler irrigation system has been the more popular method for curing RCC. Owing to the low water content in RCC, strict control on moist-curing is essential for ensuring that the desired properties are achieved. It has been found that frost resistance may be enhanced by an extended period of moist-curing. Early traffic should be restricted to light loads until the concrete is fully cured since the RCC is susceptible to microcracking and other structural damage during the early stages [10].

2.4.3/ Joints and Sawcuts

Paving in echelon, similar to the technique used in asphalt paving may be utilized. It is recommended that a 300-450 mm wide strip should be left unrolled along the longitudinal joint until the adjacent lane has been placed. This construction practice allows a good concrete density and bond to be achieved along the longitudinal joint, thereby reducing the potential for ravelling. For intermodal terminals, etc., the direction of paving should be in the shortest dimension to reduce the length of time that the open face of a longitudinal joint is left exposed and to ensure that any cold joints at the end of each day's paving are in the shortest direction. The edge of the last

lane should be scraped vertically with a grader blade back into the compacted material. When construction resumes the following day, a cement-water grout mix should be uniformly applied to the exposed hardened vertical face to ensure that an adequate bond develops between both sides of the cold joint. Existing RCC pavements in service show that the cold joint may be prone to rapid deterioration if not properly treated [10].

Sawcutting transverse joints to control cracking has not been very successful [19]. It has been found that RCC pavements constructed without sawcut control joints exhibit no serious distress and require minimum maintenance [10]. Regular shrinkage cracks in RCC pavements usually occur between 13 to 30 m, depending on the pavement thickness. Generally, the uncracked lengths are much less for conventional Portland cement concrete.

2.4.4/ Surface Tolerances

Work completed on a test road at Fort Lewis using two mixes, one with 19 mm aggregate and the other with 16 mm maximum size aggregate, consistently produced surface deviations meeting an average 5 mm tolerance when measured transversely with a straight edge [1]. The U.S. Army Corps of Engineers typically specify a maximum tolerance of 10 mm, using a 3 m straightedge, measuring transverse to centerline. In Ontario, a maximum tolerance of 3 mm is specified for conventional concrete pavements [20].

2.4.5/ Quality Control

Quality control is important for ensuring high quality RCC construction. Routine sampling of RCC is necessary to ensure the mix meets the specifications. Nuclear density meters have been successfully used for checking the in-place compacted densities. Typical compaction targets of 96% of maximum air-free theoretical density or 98 to 100% of the Modified Proctor (ASTM D1557) are recommended [10]. It is also recommended that surface tolerances should be continuously monitored using a 3-3.6 m long straightedge. Control on workability (rollability) has been attempted using a modified Vebe device [21] or a modified Kango hammer [8].

2.5/ Comparisons between Roller Compacted Concrete, Conventional Concrete and Lean Concrete Base: A Summary

The main differences between the various Portland cement concretes are the water/cement (w/c) ratio and slump, which control strength characteristics, and the manner of concrete placement, respectively. As indicated previously, conventional concretes are manufactured using a blend of durable aggregates to achieve a desired gradation, water, Portland cement and additives to control workability, air content, etc.. The resulting product is a high quality material whose strength depends mostly on the water/cement ratio which is selected according to application strength requirements; e.g., slab-on-grade, retaining wall, column, etc.. Conventional concretes generally have slumps between 15 and 75 mm. For paving applications, conventional air-entrained concretes are placed via a fixed-form or slipform paving train that places, vibrates, screeds and finishes the concrete in a smooth series of operations. Slipforming generally requires a low slump concrete (<30 mm) to ensure that the pavement retains its shape after the paver has passed (see, e.g., Reference 17).

In comparison, RCC is generally manufactured using a single well-graded durable aggregate, water, and Portland cement without additives. The fresh concrete, which has the appearance of moist gravel, has zero slump. The zero slump concrete is usually placed via an asphalt paver and is immediately compacted by a vibratory compactor to achieve the desired density, and thus the desired strength. Based on the literature, the properties of RCC and conventional concrete are similar; although RCC may have inferior fatigue strength.

Lean concrete base, as the name implies is a low cement content and thus weaker material used for base construction. It may be placed as an air-entrained slump concrete (Econocrete) using fixed or slipform pavers, or it may have a zero slump and be placed like RCC. The zero slump lean concrete base is used primarily in the United Kingdom [22] whereas Econocrete is more popular in the USA [23,24]. The purpose of using low cement content in Econocrete is to reduce reflection cracking in the asphalt surface course. However, it should be noted that the British have experienced reflection cracking problems with their lean concrete base. To reduce the severity of the problem they increased the thickness of their surface course [22]. Experience to date suggests that RCC cracks much less than lean concrete base.

3/ Laboratory Testing Programme

At the beginning of the RCC study a review of the literature clearly indicated that important design information was lacking in the areas of freeze-thaw and salt scale durability, and fatigue behaviour of roller compacted concrete. As a result, it was decided that an experimental programme should be undertaken to compare the strength and durability properties of RCC and slump concrete, which would act as a control. The fatigue testing was performed by using a split cylinder loading configuration.

The experimental programme involved comparing the performance of RCC at various cement contents to that of the control mixes prepared by using lean concrete methodology. This approach was chosen since it allowed the RCC test results to be interpreted within a wider context than if testing had been completed on RCC alone. Whenever possible, the appropriate ASTM specifications were followed. Deviations from the ASTM specifications are discussed within the following sections.

3.1 / Specimen Preparation

3.1.1/ Raw Materials

(a) Aggregates

Owing to the variability of aggregates in Ontario, it was recognized that if RCC should become a viable alternative to lean or conventional concrete, each contract would require suitable testing of the local aggregates which would be used for the nearby contract. For purposes of demonstrating the influence of aggregates on RCC properties and differences between the competing concrete technologies, tests were carried out using the aggregates listed in Table 3.

A standard sieve analysis (ASTM C-136) was performed on each aggregate. Percent passing curves are given in Figure 4. This figure shows that, with the exception of excess fines, the gradations are close to or within the standard concrete aggregate limits. In preparation for batching, each aggregate was laid out on the floor for three to four days to dry and was then weighed and stored in sealed plastic pails to prevent overdrying or moisture gain. It should be noted that all aggregates were in a natural dry state.

Table 3/ Summary of aggregates used for RCC study

Origin	Location	Designation	Previous MTO Contract
Nelson Aggregate Co.	Sulfur Springs+	Granular A	No. 86-44
Gormley Aggregates Ltd.	Brechin+	Granular B	No. 86-35
TCG Materials Ltd.	Aberfoyle*	Granular A	No. 86-53
J.C. Duff Ltd.	Limehouse*	Granular B	No. 86-44

+Crushed Limestone Aggregate

* Glacial /Fluvial Pit run Aggregate

(b) Cement

Type 10 normal Portland cement manufactured by Ciment Quebec was used in this study. A large quantity was purchased from a local supplier at the start of the project and was then stored in a low humidity environment in order to maintain material consistency throughout the testing programme. Blaine fineness tests (ASTM C-204) were performed on nine random cement samples yielding an average value of 3318 cm^2/g.

(c) Additives

Fresh supplies of air-entrainment agent (Pro Air at 40 ml/100 kg cement) and a water reducing agent (PKN at 250 ml/100 kg cement) were obtained from RED-D-MIX Concrete Company, located in Hamilton, Ontario. The additives were only used for manufacturing the control specimens. In order to maintain a reasonable control on additive dosage, syringes were used for measuring the additive quantities. The additives were subsequently mixed with the water, before being added to the cement-aggregate mixture, to ensure proper dispersion of additive throughout the fresh concrete.

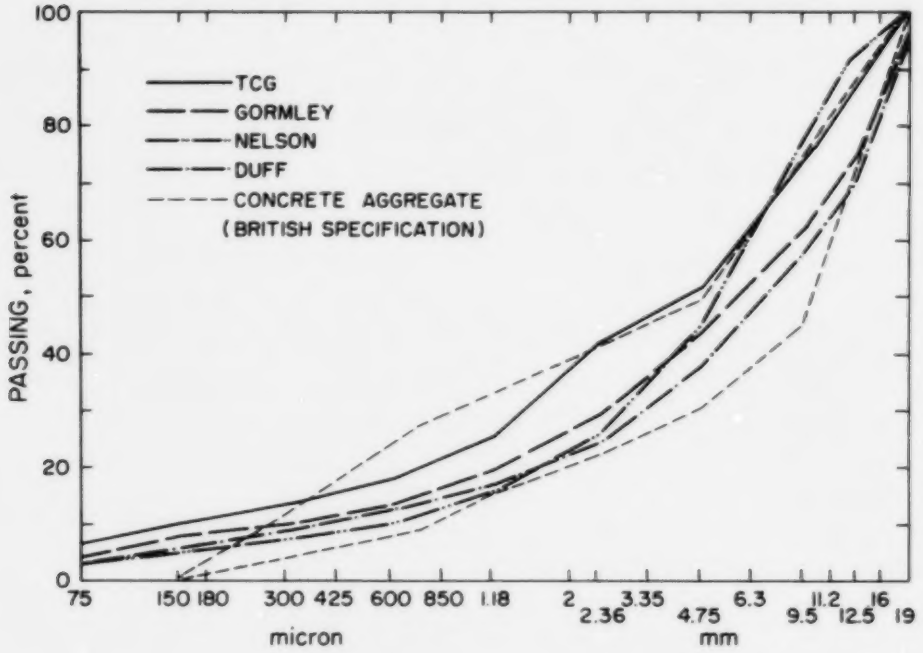


Figure 4/ Gradation of aggregates used for the RCC study

3.1.2/ Concrete Proportioning and Manufacturing

(a) RCC

Originally, the optimum moisture contents were determined by using the Modified Proctor method as discussed in the literature [6]; see ASTM D1557 for procedure. However, the examination of the hardened concrete specimens clearly indicated that suitable densities could not be achieved; i.e., in some cases the hardened concrete was very porous. It should be noted that Tayabji and Okamoto [13] also reported limitations associated with using Proctor-type or vibration table procedures for establishing the optimum moisture content of a RCC mix. Subsequently, an alternative approach borrowed from the asphalt concrete Marshall mix design procedure was adopted; see Appendix A for details on the procedure. Initially, samples were compacted at 60 blows per side; however, due to the fear of fracturing the coarse aggregate with excessive compaction, the blow count was subsequently dropped to 30 blows per side.

Optimum paste content for a given water/cement ratio was defined as the paste content corresponding to the condition where paste just begins to be squeezed out of the sample during compaction. It was found that the splitting tensile strengths of the Marshall briquettes did not vary much with a plus or minus 2% paste content variation about optimum paste content.

For Series A, the optimum paste content was found to be approximately 14% at a w/c ratio of 0.7. This w/c ratio corresponds approximately to 20 MPa strength for slump concrete. The percent paste did not vary significantly from aggregate to aggregate. For Series B the optimum percent paste content was 22% at a w/c of 0.36. At this w/c ratio, the compressive strength should exceed 30 MPa for conventional non-airentrained mixes. Table 4 presents details on the RCC mix proportions for both test series. It should be noted that an Eirich E-2 batch mixer was used to batch Series A, Group A RCC mixes. Group B and Series B specimens were batched one at a time using a small Hobart bowl mixer.

(b) Control Mixes

In contrast to the RCC mixes, the control mixes were proportioned for an approximate 50 mm slump with 6 to 8% air voids. Water/cement ratios of 0.6 and 0.36 were used for the first and second series, respectively. Table 4 presents details on the control mix proportions and properties of the concrete in a plastic state. As may be noted, problems were encountered in obtaining suitable air content in the Gormley mix for Series A. Rather than overdosing the mix, the specimens were cast at the lower air content. The Eirich mixer was used for batching all control mixes.

3.1.3/ Manufacture of Specimens

(a) RCC

A preliminary study on RCC at McMaster University [25] indicated that roller compacted concrete specimens could be molded by applying a static load to the top of the concrete and then vibrating it, (see, e.g., Reference 13). This approach for specimen preparation using a Vebe shake table was initially adopted in the study. However it was found that flexural fatigue and salt scale durability specimens of suitable density could not be manufactured using the mix designs shown in Table 4. It was later learned that the natural moisture content of the aggregate was not properly taken into account during the preliminary study. As a result, although the fresh concrete had zero slump, it contained more water and was therefore more workable than the concrete used for the study reported herein.

Table 4/ Details for mixes used in RCC study

Date	Source	Mix	A (kg)	C (kg)	W (kg)	AEA (ml)	WRA (ml)	Slump (mm)	Air (%)	Gs
SERIES A										
6-10-87	TCG	RCC	118.0	11.8	8.3					2.52
		Control	157.0	31.4	11.8	12.6	78.5	60.0	6.7	2.36
8-10-87	Nelson	RCC	88.6	7.9	5.5				2.50	
		Control	118.0	32.6	19.8	11.6	72.2	38.0	7.2	2.35
13-10-87	Duff	RCC	88.6	8.9	5.3				2.46	
		Control	133.0	22.2	11.3	8.9	55.4	57.0	6.8	2.28
15-10-87	Gormley	RCC	88.6	8.9	5.3				2.40	
		Control	95.0	22.2	13.3	8.9	55.4	57.0	3.5	2.25
SERIES B										
22-11-87	TCG	RCC	1.2	0.194	0.07					
		Control	82.2	37.800	13.60	15.1	94.7	64.0	7.0	2.48
22-11-87	Nelson	RCC	1.2	0.194	0.07				2.57	
		Control	97.6	37.800	13.60	15.1	94.7	57.0	6.5	2.43
24-11-87	Duff	RCC	1.2	0.194	0.07				2.49	
		Control	60.4	25.000	9.10	10.1	63.0	51.0	5.5	2.29
24-11-87	Gormley	RCC	1.2	0.194	0.07				2.50	
		Control	66.7	25.000	9.10	10.1	63.0	46.0	6.1	2.32

NOTE:
1. A = Aggregate; C = Cement; W = Water; Gs = Specific Gravity
   AEA = Air entraining agent; WRA = Water reducing agent
2. Series A/Group A and Series B - 20 specimens for each mix
   Series A/Group B - 12 RCC specimens for each mix
3. Each RCC specimen in Series B was mixed separately

In order to manufacture properly compacted RCC specimens, the Marshall procedure for compacting asphalt concrete briquettes was used; see Appendix A for details. Owing to the nature of this approach, the specimens were molded according to control mass (i.e., 1200 g and 2000 g of aggregate for 100 mm diameter split cylinder and freeze-thaw durability specimens, respectively). It should be noted that higher molds had to be fabricated for manufacturing the taller freeze-thaw durability specimens. The salt scale specimens were cut from extra freeze-thaw specimens to form a nominal 100 mm diameter surface.

Although the procedure for fabricating specimens deviated from those used for conventional concrete, it was felt that better specimen manufacture control could be exercised with the approach

adopted in this study, thereby ensuring better uniformity of densities between specimens of the same mix design than could have been achieved by another laboratory approach. In a recent study, it was found that flexural properties of fabricated specimens were more variable than those recorded using sawed beams from field compacted concrete [26]. As indicated previously, fatigue tests were completed using a split cylinder loading configuration.

(b) Control Concrete

The freeze-thaw durabiiity samples (150 x 100 mm diameter) were cast according to ASTM D560. The split cylinder control specimens were cast using the same molds used for fabricating the RCC specimens to ensure that the geometry of all specimens was similar (i.e., briquettes having 100 mm diameter). All specimens were vibrated on the Vebe shake table in order to achieve proper compaction.

3.2/ Durability Test Procedures

3.2.1/ Freeze-Thaw Testing

The laboratory freeze-thaw tests were performed on the RCC and control specimens following ASTM D560 "Freezing and Thawing Tests of Compacted Soil-Cement Mixtures". This method was chosen over the more severe ASTM C666 procedure "Resistance of Concrete to Rapid Freezing and Thawing". It was felt that this testing procedure was suitable since the RCC would serve as a high quality base material which would be covered by asphalt concrete, thereby eliminating direct exposure to environmental changes. It should be noted that the RCC specimens were generally shorter than recommended in the ASTM specifications; 100 to 125 mm high rather than 150 mm.

3.2.2/ Scale Resistance Testing

The scale resistance tests were carried out following ASTM C672 "Scaling Resistance of Concrete Surfaces Exposed to Deicing Chemicals". Owing to the difficulty of fabricating (250 x 250 mm) RCC salt scale specimens of suitable density as recommended in the ASTM specifications, much smaller ones, which were made by cutting the top of split cylinder-type specimens to obtain a good smooth surface, were used for both the RCC and control mixes. Two procedures were implemented for testing the effect of the salt solution on the concrete:

1. Salt water solution contained by a dyke was placed on the surface as recommended in the ASTM specification.

2. Specimens were immersed 6 mm into the salt water solution, resting on a rack.

The state of each specimen, two to three specimens for each mix, was evaluated as recommended in the ASTM procedure. Again, it was felt that the deviation from the normal practice did not in any way undermine the ability to properly evaluate the salt scale resistance of the RCC mixes relative to the control mixes since both concretes were tested identically. To assess the impact of using the smaller specimens rather than the larger ones, scale resistance tests were also completed using larger (250 x 250 mm) control specimens and two RCC specimens from Series A. All other, large-size RCC specimens were so porous that they could not retain the salt water solution; i.e., the water seeped through the 75 mm thick samples. It was found there was little difference between the small and large sample responses to the saltwater, as anticipated. However, owing to the smaller exposed surface area of the smaller specimens, it was more difficult to estimate the percent surface area that underwent deterioration than was possible for the larger ones.

3.3/ Fatigue Testing

After the split cylinder specimens were manufactured, they were cured for 28 days in the moist room and for a minimum of 10 additional days in a low moisture environment on the laboratory counter. The split cylinder tests were performed using a MTS closed-loop servo-controlled hydraulic ram system. Testing was completed in two stages for each mix:

1. Three samples were statically broken using the displacement ram mode in order to establish an average strength.

2. Samples were then failed under cyclic loading with the same support conditions used for the static tests, see Figure 5. A load control mode, using a compressive haversine wave at a frequency of 10 Hz, was required to maintain uniform cyclic loading. The applied load varied between nominal 60 and 90% of the static strength. During this stage, some samples were also failed statically to determine the significance of possible strength gain with time.

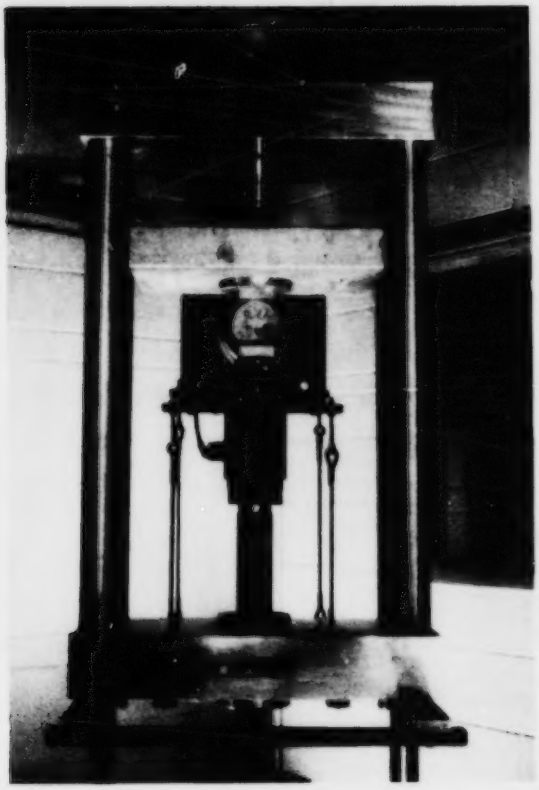


Figure 5/ Basic experimental setup for split cylinder tests

4/ Results and Discussion

4.1/ Durability Tests

4.1.1/ Freeze-Thaw Tests

The results of the freeze-thaw tests for Series A were divided into two groups -- moisture content, and mass change monitoring. All raw data are given in Appendix B. Figures 6 and 7 show the percent moisture content over 12 cycles for both the RCC and control specimens of Series A, respectively. Taking into account natural fluctuations in measured moisture contents, equilibrium moisture conditions were attained within 1 to 2 cycles. Figures 8 and 9 summarize mass change of each specimen tested over the same 12 cycle period. The change in mass, which takes into account moisture content fluctuations, was calculated according to:

$$\% \text{ Mass Change} = \frac{\% \text{ Mass Change}}{\text{Brushed Specimen}} - \frac{\% \text{ Mass Change}}{\text{Control Specimen}}$$

It is clearly shown in these figures that the mass changes were quite small (within 0.5%). The mass changes are much smaller than the maximum permissible of 14% for cement treated bases (see, e.g., Reference 17, 5 -10). A qualitative visual examination of all specimens confirmed the quantitative assessment. The texture of all specimens at the end of the test looked the same as at the beginning; thereby suggesting that little material was brushed off. It is important to note that mass changes of RCC specimens were negligible after 4 cycles; i.e., 4 cycles were required to brush off loose material generated during sample manufacture.

The cement content for the Series B was decided upon only after reviewing the results from Series A. Since it was eventually decided that Series B specimens should have a higher cement content than those of Series A, it was felt that freeze-thaw testing of the latter series was not necessary due to the excellent performance of all samples at the lower cement content and the fact that the scale resistance samples also were subjected to freeze-thaw cycles. A richer mix would be expected to have higher strength and thus be more durable than a leaner one. Although freeze-thaw tests were not completed on specimens for Series B mixes, a few freeze-thaw tests were completed on leaner RCC mixes. The results from these tests are similar to the ones shown in Figure 8.

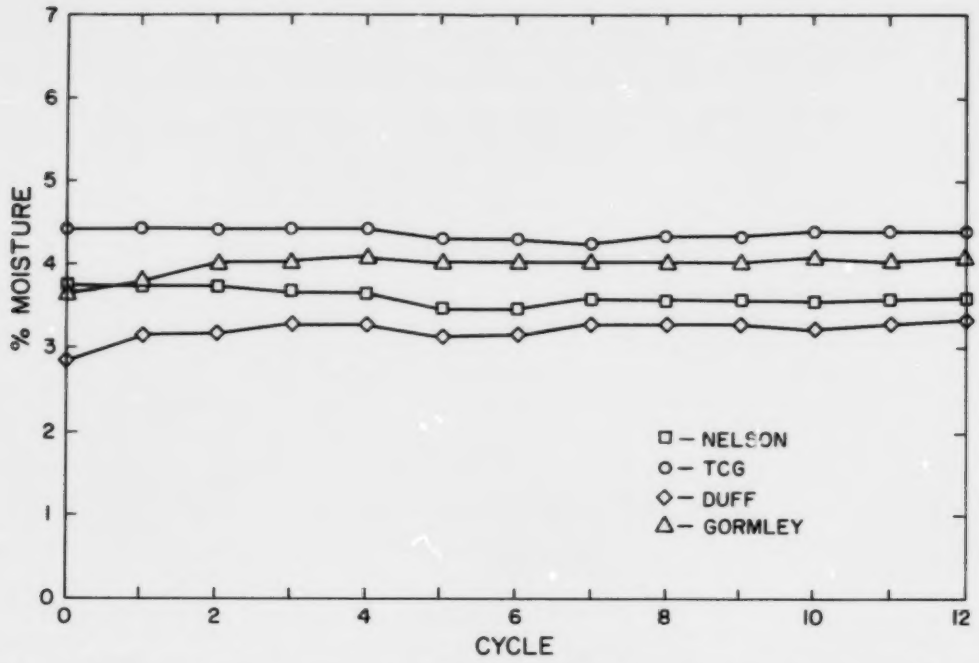


Figure 6/ Percent moisture for RCC specimens during freeze-thaw test (Series A)

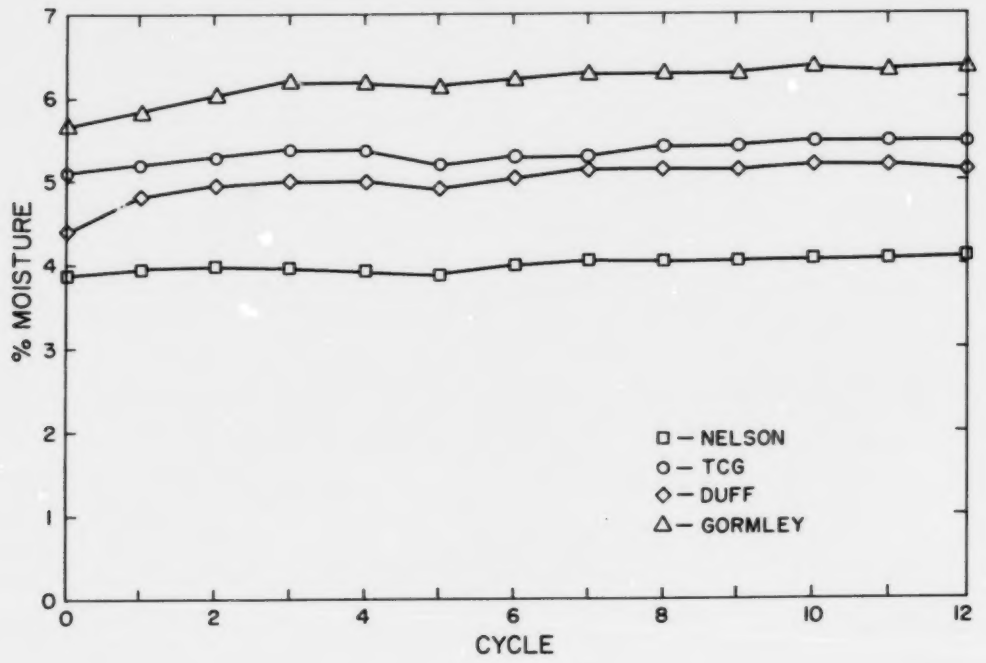


Figure 7/ Percent moisture for control specimens during freeze-thaw test (Series A)

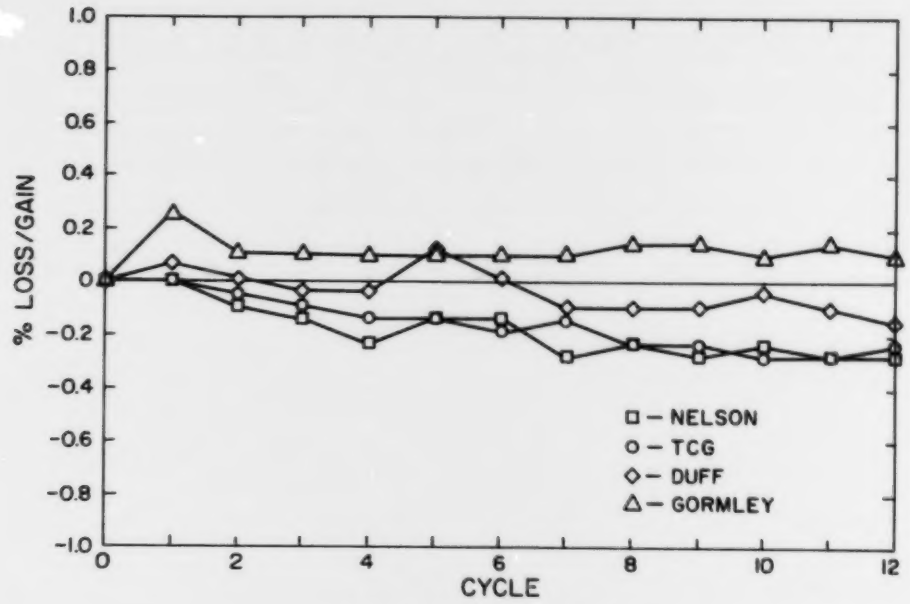


Figure 8/ Percent mass change of brushed RCC specimens during freeze-thaw test (Series A)

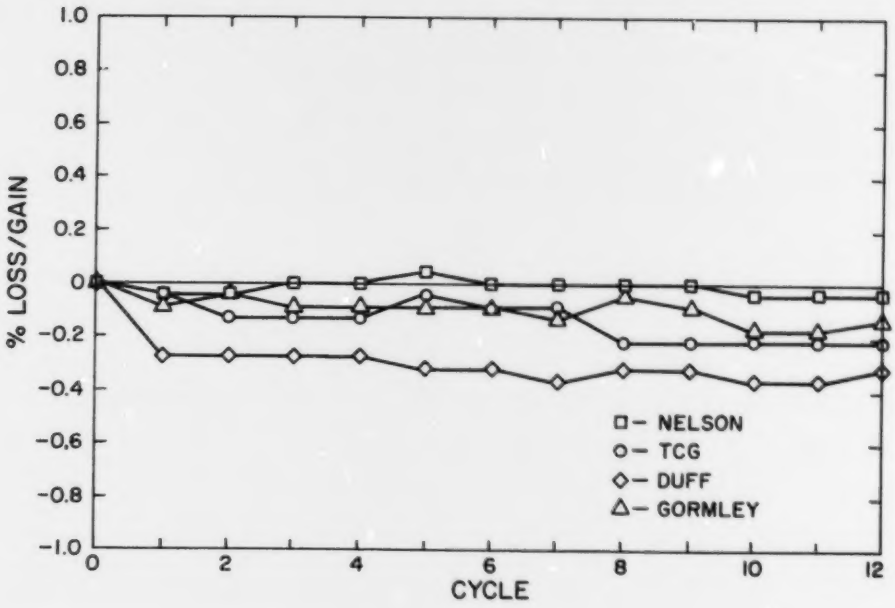


Figure 9/ Percent mass change for brushed control specimens during freeze-thaw test, (Series A)

4.1.2/ Scaling Resistance Tests

Figures 10 through 13 show the state of selected specimens at the end of the salt scale testing. See Appendix C for tables giving all quantitative surface area deterioration assessments. The observations made during this phase of the experimental testing programme may be summarized as follows:

1. Specimens made with Gormley aggregate were poor regardless of evaluation procedure. It was clear that the properties of this particular aggregate contributed to the the poor performance.

2. Control specimens exhibited better scale resistance than RCC specimens.

3. Small-dyked specimens exhibited lower resistance than large-dyked ones. This has been attributed to the differences in the nature of the surfaces. For large specimens paste coated the entire exposed surface, thereby providing extra protection. The small cut specimens did not have such a protective coating, which allowed direct salt attack at paste-aggregate interfaces.

4. Based on small dyked specimens, scaling resistance at lower cement content was generally poor.

5. Based on small dyked specimens, scaling resistance at higher cement ranged from reasonable to good.

6. All immersed specimens experienced salt attack at the waterline.

These observations were based on the visual assessment of the group response by placing all specimens together and not individual specimen responses. The quantitative assessment given in Appendix C is based on visually estimating the percent deteriorated surface area, which was also subjective, of individual specimens. Care must be taken when quantitatively interpreting these results, particularly for the small specimens. It may be said, however, that the mixes of Series B containing the Nelson performed the best, and the TCG and Duff aggregate mixes experienced a low level of salt attack, whereas the Gormley mixes completely deteriorated within 10 cycles.

Although all specimens showed some sign of salt attack, it is felt that, from a durability viewpoint, RCC is a viable base material provided that a durable aggregate is used, proper compaction is achieved and sufficient paste is available to coat the aggregates; i.e. high cement content mix (13-14%). It should be noted that these findings are conistent with those of Andersson [7,8].

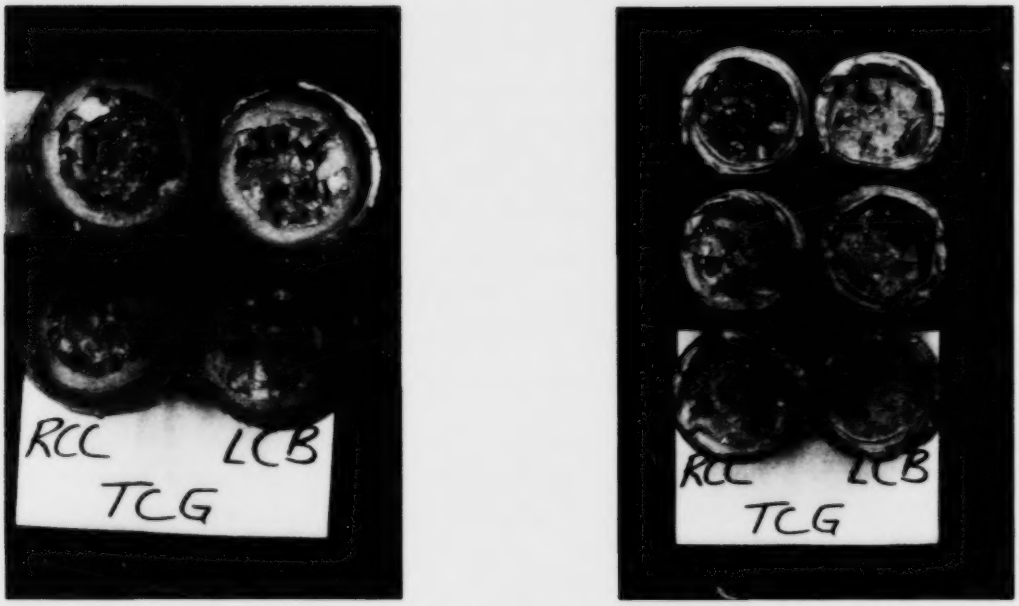
4.2/ Split Cylinder Strengths

4.2.1/ Static Tests

The experimental results indicated that the RCC specimens were, on average, stronger than the control specimens; as shown in Table 5. An increase in cement content resulted in a corresponding increase in static split cylinder strength for RCC. It was felt that the increase in strength of the RCC mixes due to higher cement content could be attributed to both the decrease in w/c ratio and the increase in optimum paste content of Series B relative to Series A. The decrease in strength with increased cement content of the control concrete for two of the aggregates is still not clear. Coarse aggregate fracture (~95%) dominated over debonding for most specimens of Series A and B. However, it should be noted that in some cases the lower strength of a specimen could be attributed to the presence of flaws along the failure plane; e.g., lack of paste between large aggregates or presence of trapped air pockets. No firm conclusions could be made regarding strength gain with time since strengths at later dates were generally similar to those established at the beginning of this phase of the testing programme and the scatter in strengths generally undermined the detection of possible strength gains.

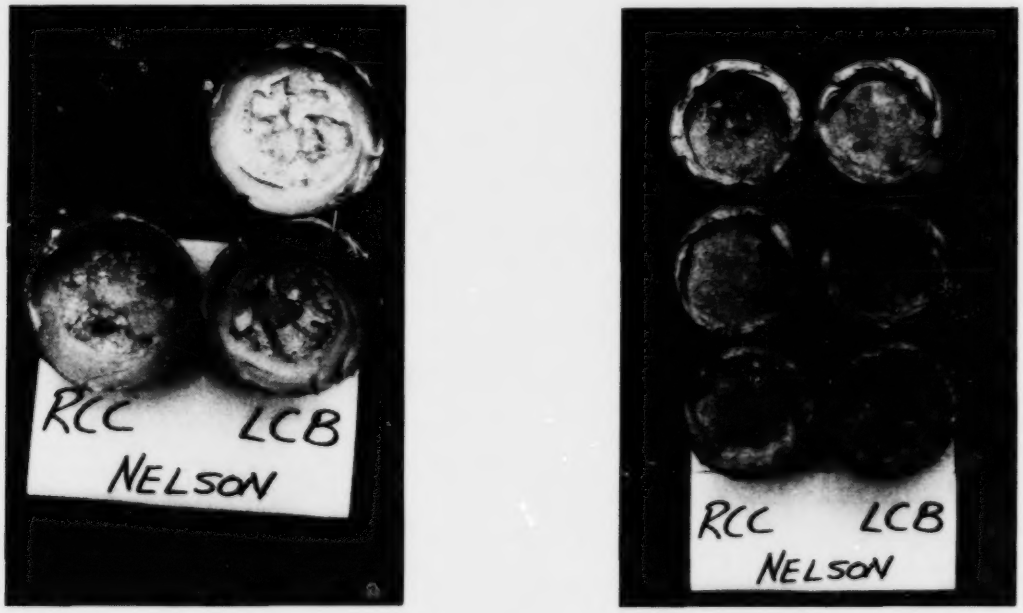
Table 5/ Static Split Cylinder Strengths (MPa)

Type	Test Series A		Test Series B	
	RCC	Control	RCC	Control
TCG	4.52	3.96	5.86	4.24
Nelson	4.81	4.99	4.92	4.53
Duff	4.79	3.87	5.30	4.42
Gormley	2.84	3.36	3.73	3.12



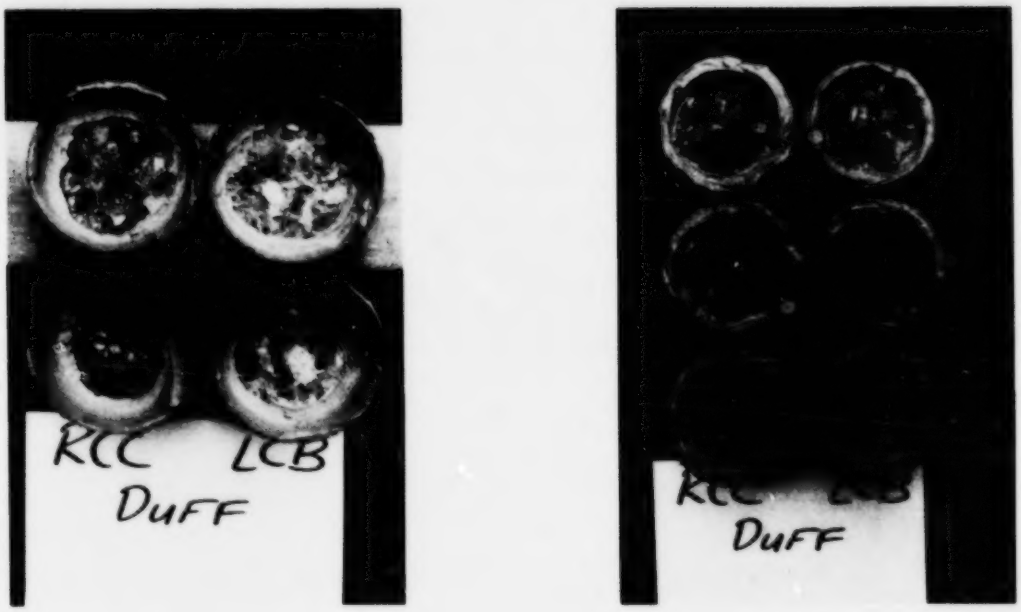
(a) (b)

Figure 10/ Scale resistance of TCG mix; (a) Series A; and (b) Series B



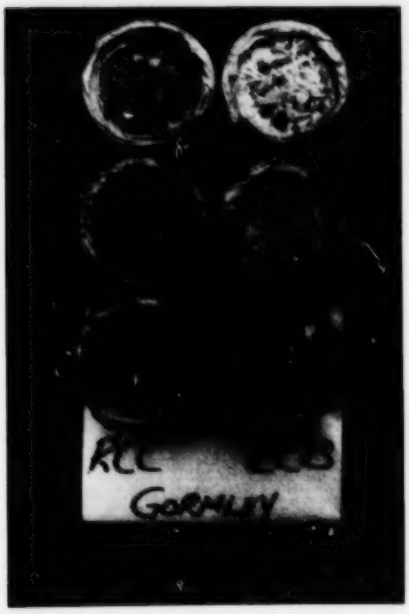
(a) (b)

Figure 11/ Scale resistance of Nelson mix; (a) Series A; and (b) Series B



(a) (b)

Figure 12/ Scale resistance of Duff mix; (a) Series A; and (b) Series B



(a)

Figure 13/ Scale resistance of Gormley mix (Series B)

A correlation between compressive f'_c and split cylinder f_t strengths was established in a separate unrelated study. Tests completed on 7- and 28-day old specimens showed the following relationship:

$$K = \frac{f_t\,(MPA)}{\sqrt{f'_c}} = 0.55$$

with a coefficient of variation of 1.6%. Since the modulus of rupture M_r is approximately 50% greater than f_t (see, e.g., Reference 6, Chapter 13), the flexural strength for third point loading is approximately:

$$M_r = 0.82\sqrt{f'_c}\ (MPa) = 9.9\sqrt{f'_c}\ (psi)$$

which gives flexural strengths that are higher than the minimum values predicted using the relationship for RCC from the PCA [15].

4.2.2/ Fatigue Tests

The results of the fatigue tests are shown in Figures 14 through 17, and are given in tabular form in Appendix D. A summary of typical elastic properties are also given in this Appendix. The scatter shown in these curves is not unusual and was expected since small specimens were tested (see, e.g., Figure 8.25 in Reference 27). Unfortunately, the scatter made it difficult to interpret the data. There are two sources for the scatter shown:

a) two specimens with identical static strength may have different fatigue strengths, and

b) static strength in the fatigue sample may be different from that assumed.

To the first order, the variation in stress ratio SR may be estimated by:

$$SR = \overline{SR}\ (1 \pm x)$$

where SR is the average stress ratio based on static strengths and x is the variation in static strength. Since the static strength of each fatigue sample was not known, it was decided that a point on the SR versus log(N) plot should be included for each static strength; i.e. if n static strengths for a particular mix design were obtained, then n possible stress ratios for each fatigue test were generated. This data interpretation technique gave a better picture on the distribution of

SR versus log (N) possibilities than would have been possible using the mean responses alone, since the effect of the scatter in the static results appeared explicitly in the fatigue data. It should be noted that, while some specimens were tested beyond 1 to 2 million cycles, others were failed statically if the test took too long to complete. It was found that the post cyclic-loading static strength for these specimens was often much higher than that which had been assumed based on the earlier static tests.

Based on the results of Series B, this study tends to support the finding of Tayabji and Okamoto [13] that the fatigue strength of RCC is less than that of slump concrete even though the static strengths are the same or higher. Owing to the scatter, it is not possible to quantify the difference in fatigue strengths. A comparison of the data with the design curve from Reference [15] indicates that at lower stress ratios the design values are conservative whereas at higher ratios they "may be" unconservative. The lower fatigue strength of RCC when compared with that of conventional concrete may be attributed to its lower paste content and the greater sensitivity of fatigue strengths relative to static strengths to flaws in structure; i.e., at the lower paste contents, there is a greater chance for improper aggregate coating and thus a greater chance of developing flaws which reduce fatigue strength. It should be noted that the fatigue strengths of both RCC and control specimens for Series A appeared to be similar.

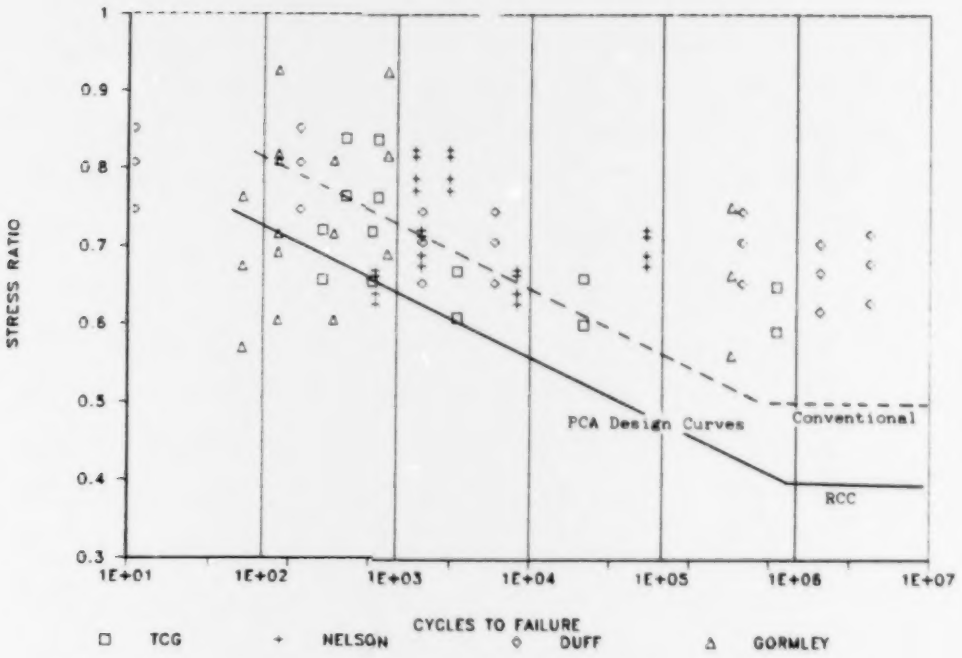


Figure 14/ Fatigue data for RCC of Series A, Group B

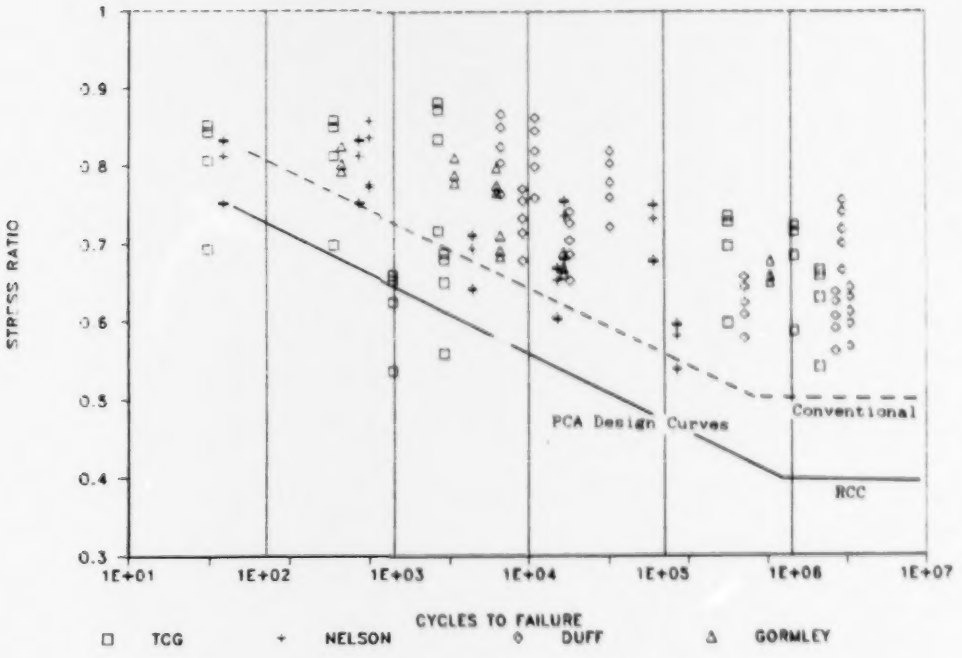


Figure 15/ Fatigue data for control concrete of Series A

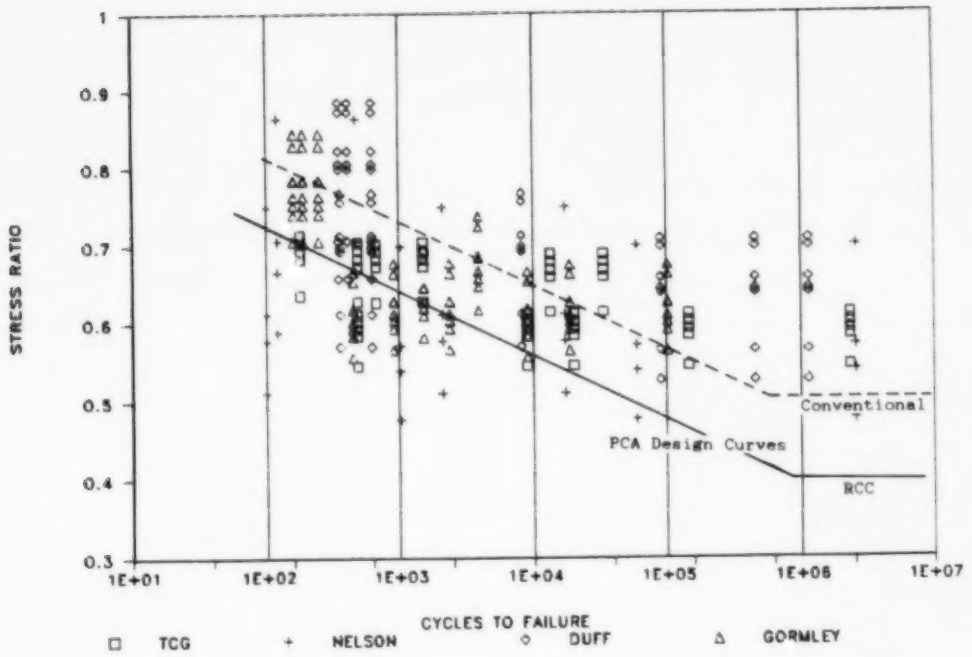


Figure 16/ Fatigue data for RCC, Series B

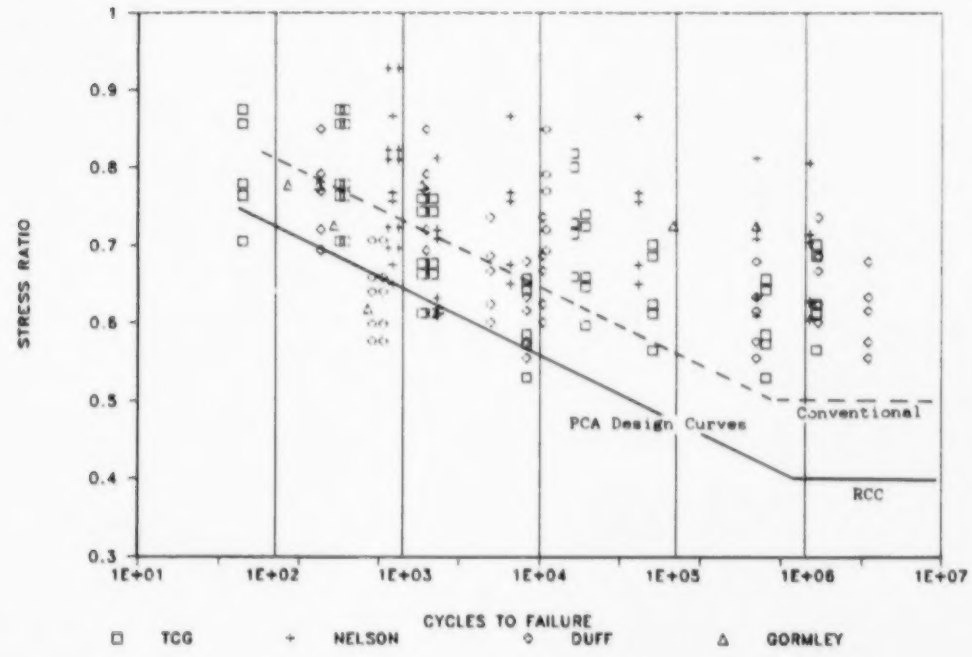


Figure 17/ Fatigue data for control concrete, Series B

5/ Comparison of RCC and Conventional Concrete Thickness Design

An important component of pavement thickness design, which applies to both RCC and conventional concrete pavements design, is fatigue analysis. Given the findings on fatigue performance from this study and others [13], it is clear that an RCC pavement should be thicker than the corresponding conventional pavement. Based on the calculations shown in Appendix E, it appears that RCC pavements must be 10 to 20% thicker than conventional concrete pavements. Of course, it must be recognized that pavement design is sensitive to volume and distribution of traffic, and other factors such as durability. Thus one should not assume *a priori* that all RCC pavements must be 10 to 20% thicker.

Supporters of RCC in the past have claimed that RCC construction costs are approximately 30% less than the competing conventional construction costs. However, owing to thicker RCC pavement and the need for an asphalt overlay for higher speed facilities, an overall 30% cost reduction cannot be realized. A "back-of-the-envelope" type calculation suggests that the potential cost savings may be no more than 3 to 4%.

6/ Material and Construction Specifications

Although RCC construction does not appear to be more economical than slipform construction, it does provide an alternative to the conventional construction practice. The following specifications are intended as guidelines for updating OPSS 350 "Construction Specification for Concrete Pavement, Concrete Base and Lean Concrete Base". These guidelines were developed using References [7] and [22].

6.1/ Material

(a) Aggregates: The gradation limits should be the same as those for a Granular A material. Maximum aggregate size should not exceed 22 mm and it is preferable to have more than 5% but less than 10% nonplastic fines passing the 0.074 mm (#200) sieve in order to reduce segregation. The aggregate must be durable and should be free of any undesirable contaminants which inhibit desirable concrete properties.

(b) Cement Content: The cement content should be selected to ensure minimum 28 day compressive and split cylinder strengths of 36 and 4.9 MPa, respectively. To ensure adequate durability the cement content should not be less than 300 kg/m^3 (approximately 14% by weight).

(c) Water Content: Water content must be selected to ensure proper hydration and maximize compacted density. It is suggested that the Marshall procedure be adopted to establish optimum water content (4.8 - 5.6% by weight).

(d) Additives: No additives are to be used.

6.2/ Construction

(a) Manufacturing: Concrete may be batched using a ready mix plant or pugmill-type soil cement mixer. Care must be taken to ensure proper mixing action. This may require cutting batch quantities to 2/3 of the capacity of the mixer.

(b) Transportation: The freshly mixed material must be protected from direct exposure to the environment to minimize moisture loss or gain during transportation.

(c) Placement: High quality RCC should be placed by an asphalt paver. Precompacted thickness should be 10 to 20% greater than the final thickness to account for the effect of compaction. A single lift should not exceed 275 mm and be preferably less than 150 mm. The RCC should be placed within an hour after mixing. Exposed longitudinal joints should be kept to a minimum by restricting paving length when using a single paver in order to prevent a longitudinal cold joint from forming.

(d) Compaction: RCC should be compacted via a vibratory roller immediately after it has been placed. To obtain best compaction results and surface tolerences, the first two passes should be completed without vibration, 6 passes should follow with vibration, and a final additional pass should then follow without vibration to remove roller marks from the exposed concrete. Final compaction should be completed within 2 hours after mixing.

(e) Joints and Sawcuts: Generally, RCC pavement should not require joints, nor should sawcuts be provided. Should a cold joint be necessary due to construction procedure, then the explosed surface should be cut back to form a vertical surface using a grader blade, after compaction is complete. A cold joint should be treated with fresh cement paste when placing fresh concrete adjacent to it, in order to form a proper bond between fresh and hardened concrete.

(f) Curing: Surface should be kept moist with sprinkler line or water truck to ensure proper hydration of concrete. After sufficient sprinkling, an asphalt emulsion seal may be applied to concrete surface to prevent loss of moisture.

(g) Tolerance and Surface Texture: For base material, the allowable tolerance in thickness should be less than 10 mm. The surface should have sufficient texture to allow proper no-slip bonding between the concrete and the asphalt concrete surface course.

(h) Control: The rollability of the concrete can be monitered by using the Marshall compaction procedure described in Appendix A. The field density must exceed 95% of the theoretical dry density at zero air voids for given water and cement content. Field densities should be monitored using a nuclear device.

7/ Concluding Remarks and Recommendations

This study provided a brief state-of-the-art on roller compacted concrete and investigated two areas of more limited experience; i.e. durability and fatigue strength which have a profound influence on the design of RCC pavements in an Ontario environment. The main conclusion drawn from this study is that RCC may be an economically feasible alternative to conventional concrete base construction. However, it has been shown that the potential cost savings reported in the literature are greatly exaggerated, partly due to the fact that RCC fatigue strengths have been overestimated in the past. Owing to the lower fatigue strengths, RCC pavements must be thicker.

Since it has only been in recent years that RCC has emerged as a competing technology, standards pertaining to specimen manufacture, testing procedures, and test result interpretation, etc. are still being developed or refinded; see, for example, Reference [6]. It is clear that existing conventional concrete or soil cement methodologies are not entirely suitable for RCC. Owing to the nature of RCC, it is difficult to consistently manufacture RCC specimens with a small variation in properties [26]. A consistent approach based on the Marshall procedure for mix design and specimen manufacture was developed and adopted for this study since it was felt that the other methodologies did not provide sufficient control with respect to sample manufacture. However, the approach advocated herein does require refinement. Ontario Hydro has recently developed a procedure for manufacturing 150 mm diameter high density cylinders by using a steel mold and small jumping-jack fitted with a 150 mm diameter end plate which can be used to make larger and hopefully less variable split cylinders for compressing test samples.

The next phases of RCC development in Ontario should involve:

1. Refinement of mix design procedure; e.g. determine optimum paste content with respect to saturated surface dry conditions rather than dry conditions.

2. Further evaluation of scale resistance of RCC using larger surface areas. The RCC specimens should be saw cut from *in situ* test strips.

3. Evaluation of the long term performance of both exposed and nonexposed RCC test strips.

Although the laboratory testing programme was very valuable for providing experience with RCC and preliminary information on RCC properties (relative to conventional mixes), a better

assessment for RCC performance can best be established through a larger scale *in situ* testing programme.

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Appendix A

A.1/ Procedures for Concrete Proportioning & RCC Fabrication

The mix design philosophy proposed herein is similar to that of conventional concrete. A suitable water/cement ratio is selected based on strength requirements and the paste/aggregate ratio is adjusted to meet minimum compactibility (workability) requirements. The compactive effort used in the laboratory should reflect that used in the field.

A.1.1/ Proportioning of Concrete

(a) Equipment: Marshall hammer and molds (100 mm diameter), Hobart mixer, scale, 100 ml or 250 ml graduated cylinder, spatula, large spoon and mixing bowl.

(b) Procedure:

1. Prepare molds by applying a thin coat of form stripping agent to inside surface and then position in compaction apparatus.

2. Weigh out 1200 g of properly air-dried aggregate.

3. For selected w/c ratio and assumed paste content, weigh out required quantity of water and cement.

4. Place aggregate and cement in mixing bowl, then dry mix for approximately 10 seconds using Hobart mixer.

5. Add water to aggregate and cement and then continue mixing for an additional 50 seconds.

6. Remove bowl from mixer, use spoon to scrape off material on sides and mix with spoon to ensure uniformity. Remove aggregate sizes greater than 16 mm nominal.

7. Add mixture to mold making sure that material is as uniform as possible; i.e. avoid segregating mix.

8. After filling mold, rod fresh mixture with spatula 3-4 times.

9. Position hammer in place and compact manually for the first two blows.

10. Finish compacting by applying 30 blows/side.

11. Repeat steps 1 through 10 at various paste contents, typically 14, 16, 18, 20, 22%.

Optimum water content by definition is the water content corresponding to the state when paste just begins to be squeezed up between the hammer head and the mold.

A.1.2/ Specimen Fabrication for Split Cylinder

(a) Equipment: Same as for Section A.1.1., hydraulic jack

(b) Procedure:

1. Follow steps 1 through 10 of A.1.1 for combination of materials corresponding to optimum conditions.

2. After compaction, cover molds with moist burlap and plastic to prevent drying of briquettes.

3. Remove briquette from molds on following day using a hydraulic jack.

4. Place briquettes in moist room for curing.

Samples manufactured in this manner are 100 mm diameter and approximately 65 - 75 mm high.

A.1.3/ Specimen Manufacture for Compression Cylinders or Freeze-Thaw Testing

Equipment and procedure are the same as for Section A.1.2, except 2000 g rather than 1200 g of aggregate should be used. Specimens should be between 100 and 125 mm high.

Appendix B

B.1/ Summary of Freeze-Thaw Test Results

Table B.1/ RCC Brushed Sample Results (Series A, Set 1)

	Mass (g)			
Cycle #	Nelson	TCG	Duff	Gormley
0	2172	2178	1772	2253
1	2172	2179	1778	2262
2	2170	2177	1778	2263
3	2168	2177	1779	2264
4	2165	2176	1779	2265
5	2163	2173	1779	2264
6	2163	2172	1778	2264
7	2163	2172	1778	2264
8	2163	2172	1778	2265
9	2163	2172	1778	2265
10	2163	2172	1778	2265
11	2163	2172	1778	2265
12	2163	2173	1778	2265
Oven Dry	2084	2078	1721	2179

Table B.2/ RCC Moisture Sample Results (Series A, Set 1)

	Mass (g)			
Cycle #	Nelson	TCG	Duff	Gormley
0	2161	2232	1851	2078
1	2161	2233	1856	2081
2	2161	2232	1857	2085
3	2160	2233	1859	2086
4	2159	2233	1859	2087
5	2155	2230	1856	2086
6	2155	2230	1857	2086
7	2158	2229	1859	2086
8	2157	2231	1859	2086
9	2158	2231	1859	2086
10	2157	2232	1858	2087
11	2158	2232	1859	2086
12	2158	2232	1860	2087
Oven Dry	2083	2138	1800	2005

Table B.3/ Control Brushed Sample Results (Series A, Set 1)

	Mass (g)			
Cycle #	Nelson	TCG	Duff	Gormley
0	2328	2278	2220	2245
1	2328	2279	2223	2246
2	2329	2279	2226	2252
3	2330	2281	2227	2254
4	2329	2281	2227	2254
5	2329	2279	2224	2254
6	2330	2280	2227	2255
7	2331	2280	2228	2255
8	2331	2280	2229	2257
9	2331	2280	2229	2256
10	2331	2281	2229	2256
11	2331	2281	2229	2255
12	2331	2281	2229	2257
Oven Dry	2237	2163	2121	2125

Table B.4/ Control Moisture Sample Results (Series A, Set 1)

	Mass (g)			
Cycle #	Nelson	TCG	Duff	Gormley
0	2327	2271	2193	2241
1	2328	2273	2202	2244
2	2329	2275	2205	2249
3	2329	2277	2206	2252
4	2328	2277	2206	2252
5	2327	2273	2204	2251
6	2329	2275	2207	2253
7	2330	2275	2209	2254
8	2330	2278	2209	2254
9	2330	2278	2209	2254
10	2331	2279	2210	2256
11	2331	2279	2210	2255
12	2331	2279	2209	2256
Oven Dry	2240	2161	2101	2121

Table B.5/ RCC Brushed Sample Results (Series A, Set 2)

	Mass (g)			
Cycle #	Nelson	TCG	Duff	Gormley
0	2296	2156	1910	2040
3	2294	2154	1908	2041
5	2290	2152	1908	2039
7	2290	2151	1908	2039
9	2291	2151	1908	2039
12	2291	2154	1908	2040

Table B.6/ Control Brushed Sample Results (Series A, Set 2)

	Mass (g)			
Cycle #	Nelson	TCG	Duff	Gormley
0	2203	2260	2238	2289
3	2208	2258	2234	2289
5	2211	2256	2234	2287
7	2210	2256	2234	2287
9	2210	2256	2234	2290
12	2214	2262	2238	2296

Appendix C

C.1/ Summary of Scale Resistance Test Results

Table C.1/ Scale Resistance Results (Series A)
(250 mm x 250 mm specimens)

		% Deterioration	
Aggregate	Cycle	Control	RCC
Nelson	5	5	30
	10	5	80
	15	5	95
	25	10	100
	50	10	100
TCG	5	5	40
	10	10	50
	15	15	75
	25	25	90
	50	80	100
Duff	5	5	
	10	20	
	15	10	
	25	20	
	50	20	
Gormley	5	100	
	10	100	
	15	100	
	25	100	
	50	100	

Table C.2/ Scale Resistance Results (Series A) (100 mm diameter specimens)

		% Deterioration			
		Dyked		Immersed	
Aggregate	Cycle	RCC	Control	RCC	Control
Nelson	5	0	0	0	0
	10	5	0	0	0
	15	5	0	15	5
	25	10	5	20	5
	50	10	5	20	5
TCG	5	0	0	0	0
	10	50	45	0	0
	15	100	45	40	10
	25	100	52.5	100	25
	50	100	60	100	25
Duff	5	0	0	0	0
	10	75	30	60	0
	15	75	76.5	100	10
	25	92.5	75	100	15
	50	100	90	100	15
Gormley	5	--	--	15	15
	10	--	--	100	100
	15	--	--	100	100
	25	--	--	100	100

Note:

1. Results for dyked samples (Procedure 1) are based on the average of two samples.
2. Results for immersed samples (Procedure 2) are based on one sample.

Table C.3/ Scale Resistance Results (Series B)
(100 mm diameter Specimens)

		% Deterioration			
		Dyked		Immersed	
Aggregate	Cycle	RCC	Control	RCC	Control
Nelson	5	2	0	0	0
	10	7	0	0	0
	15	7	3	5	1
	25	10	3	8	1
TCG	5	2	0	3	0
	10	20	5	4	0
	15	28	5	18	3
	25	28	8	30	4
Duff	5	5	0	12	0
	10	13	8	43	0
	15	27	12	62	8
	25	27	28	80	23
Gormley	5	80	10	100	20
	10	100	50	100	72
	15	100	70	100	100
	25	100	87	100	100

Note: Results of all samples are based on an average of three samples.

Appendix D

D.1/ Summary of Split Cylinder Test Results

Table D.1/ Test Series A, Group A - RCC

Aggregate	Static Failure Data			Fatigue Failure Data		
	W (mm)	P (kg)	f_t (MPa)	N	W (mm)	P (kg)
TCG	67.8	4440	4.02	500	67.7	3300
	65.5	3900	3.66	89	67	3300
	68.1	4220	3.81	26	67	3300
	68	6500	5.87	980100	66.5	2900
	67	5700	5.23	1589130	66	2900
AVG			4.52	15160	66.4	2900
				60	67.5	4525
				1616890	67	3620
				397730	67	3620
				4632200	68	3620
Nelson	71.3	5000	4.31	240	69	4310
	63	4980	4.86	140	69.6	4310
	63	4860	4.74	1970	74.6	4310
	70	6100	5.35	470	70	4632
	69	5400	4.81	233470	74	4590
AVG			4.81	6110	69	3600
				109580	70	3600
				120320	69	3600
				4827878	70	3100
				1761500	70	3760
Duff	58.5	4400	4.62	300	56.50	3500
	58	4760	5.04	20	59	3500
	59.7	4180	4.30	270	58	3500
	58	4800	5.08	106270	58.2	3060
	57	4560	4.91	3531090	58.5	3060
AVG			4.79	632260	57.5	3060
				3654259	59	3300
Gormley	63	2640	2.57	1410	63	2160
	68	2920	2.64	1760	63	2160
	68	3020	2.73	11790	63	2160
	64	3560	3.42	4376449	63	1900
AVG			2.84			

Note: W = height of sample

P = load on sample

N = cycles to failure

AVG = average strength

Table D.2/ Test Series A, Group B - RCC

Aggregate	Static Failure Data			Fatigue Failure Data		
	W (mm)	P (kg)	f_t (MPa)	N	W (mm)	P (kg)
TCG	67	5625	5.16	420	70	4926
	66	6075	5.66	720	70	4926
	67	5625	5.16	280	67	4054
	66	6075	5.66	650	67	4054
AVG			5.41	2830	67	3765
				25500	68	3767
				712810	69	3767
Nelson	67	5800	5.32	1360	66	4414
	65	5325	5.03	2440	66	4414
	66	5600	5.21	1510	66	3863
	66	5350	4.98	74840	66	3863
AVG			5.14	7860	66	3587
				690	67	3641
Duff	66	5350	4.98	190	68	4450
	68	5225	4.72	11	68	4450
	68	5950	5.38	1530	68	3894
AVG			5.03	390380	67	3837
				5410	68	3894
				3525750	68	3750
				1508190	68	3683
Gormley	69	2650	2.36	130	69	2453
	69	3550	3.16	850	69	2417
	69	3000	2.67	130	68	2146
AVG			2.73	340	69	2146
				323450	69	1993
				70	68	1993

Table D.3/ Test Series B - RCC

Aggregate	Static Failure Data			Fatigue Failure Data		
	W (mm)	P (kg)	f_t (MPa)	N	W (mm)	P (kg)
TCG	72	6600	5.63	180	71	4644
	71	6600	5.71	490	72	4650
	71	6800	5.88	1510	72	4650
	72	6800	5.80	670	72	4644
	72	6800	5.80	9030	72	4037
	73	7500	6.31	19950	72	4037
AVG			5.86	490	72	4037
				145670	72	4037
				2345850	72	4037
				33340	71	4487
				13460	71	4487
Nelson	70	4500	3.95	2120	70	3375
	70	6600	5.79	100	70	3375
	68	5350	4.83	17300	70	3375
	69	5750	5.12	1030	70	3150
AVG			4.92	2572000	70	3150
				59260	70	3150
				470	70	3890
				120	71	3946
Duff	70	5425	4.76	350	70	4739
	72	7400	6.31	620	70	4739
	70	5925	5.20	410	70	4739
	71	6800	5.88	8200	70	4107
	69	5275	4.70	620	70	4107
	72	6050	5.16	360	70	4107
	69	5675	5.05	89450	68	3683
AVG			5.30	1143800	68	3683
				460310	68	3683
Gormley	69	4125	3.67	160	73	3425
	70	4300	3.77	250	73	3425
	70	4650	4.08	190	73	3425
	73	4550	3.83	101390	68	2552
	72	4000	3.41	18800	69	2590
	73	4625	3.89	450	70	2590
	71	4250	3.68	9090	70	2590
	72	4075	3.48	1530	71	2748
AVG			3.73	2390	73	2748
				910	73	2748
				3920	67	2748

Table D.4/ Test Series A - Control

Aggregate	Static Failure Data			Fatigue Failure Data		
	W (mm)	P (kg)	f_t (MPa)	N	W (mm)	P (kg)
TCG	52	3840	4.54	2064	57	3020
	62	3760	3.73	337	58.6	3020
	59	3740	3.89	38	59	3020
	55	3300	3.69	2283	61	2520
AVG			3.96	1000000	58	2520
				933	63.5	2520
				314616	57	2520
				1567360	63	2520
Nelson	64	5500	5.28	630	70	4650
	63.3	4900	4.76	50	72	4650
	64.2	5500	5.26	530	72	4650
	68	5400	4.88	3750	72.5	4000
	67	5200	4.77	18570	68.3	4000
AVG			4.99	86320	67	3900
				129600	67	3100
				16490	65.5	3400
Duff	52	3260	3.85	6210	51	2640
	51	3280	3.95	11230	51.3	2640
	52	3520	4.16	41170	54	2640
	53	3160	3.66	9040	50	2300
	50	3040	3.74	2323219	51	2300
AVG			3.87	20530	52	2300
				418620	50	1960
				2057276	51.5	1960
				2643700	51	1960
Gormley	63.3	3380	3.28	390	59	2600
	63	3460	3.37	5750	61	2600
	60.5	3360	3.41	2770	60	2600
AVG			3.36	661980	63	2280
				6120	60	2280
				18430	62	2280

Table D.5/ Test Series B - Control

Aggregate	Static Failure Data			Fatigue Failure Data		
	W (mm)	P (kg) f_t	(MPa)	N	W (mm)	P (kg)
TCG	75	5800	4.75	320	77	4200
	75	5250	4.30	60	77	4200
	76	5425	4.39	350	77	4200
	75	4675	3.83	18000	76	3878
	73	4650	3.91	1610	75	3554
AVG			4.24	1310	75	3554
				21850	78	3600
				7920	76	3112
				481790	76	3112
				69800	76	3324
				1154070	76	3324
Nelson	77	6100	4.87	890	76	4355
	75	5300	4.34	740	76	4355
	77	6100	4.87	54720	76	4065
	74	5150	4.28	6000	76	4065
	79	4875	3.79	800	76	4065
	76	6250	5.05	1024240	73	3631
AVG			4.53	412220	77	3860
				1730	77	3860
Duff	78	5550	4.37	230	76	4163
	77	5325	4.25	1430	76	4163
	76	6000	4.85	11160	76	4163
	76	4900	3.96	10540	77	3656
	72	5475	4.67	1185540	77	3656
AVG			4.42	4270	77	3656
				410030	75	3287
				2811250	77	3374
				7910	77	3374
				560	78	3554
				680	74	3371
Gormley	76	3375	2.73	130	76	3003
	77	4600	3.67	1320	76	3003
	77	3750	2.99	220	76	3003
	78	4500	3.54	99080	76	2802
	76	3800	3.07	290	76	2802
	74	3675	3.05	408840	76	2802
	77	3500	2.79	1780	78	2450
AVG			3.12	520	78	2450

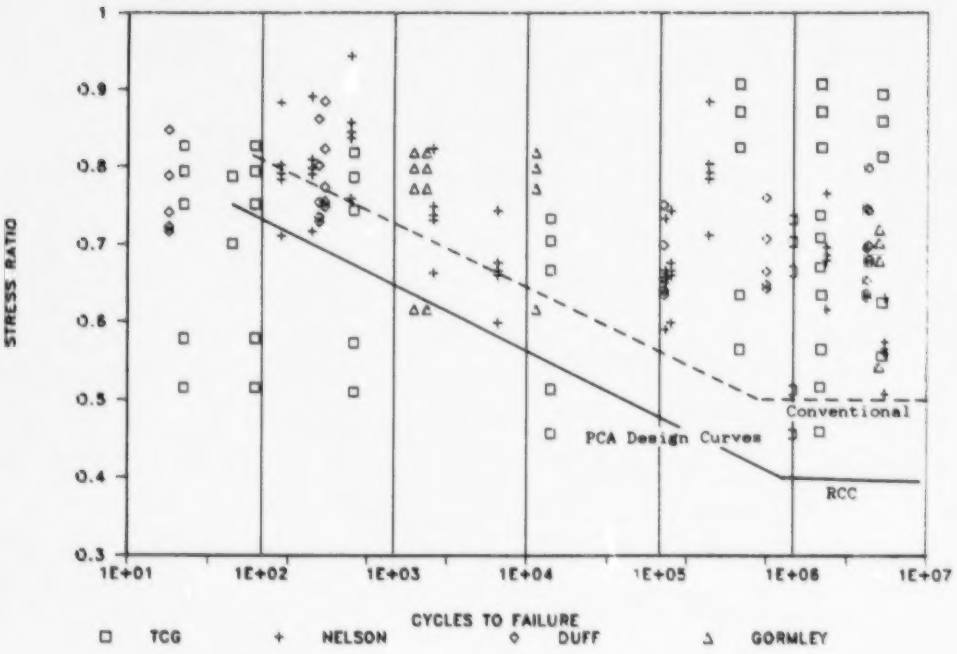

Figure D.1/ Stress ratio vs. cycles to failure, Test Series A, Group A - RCC

Appendix E

E.1/ Comparison of RCC and Conventional Pavement Designs

E.1.1/ Structural Design

A comparison of pavement designs between the two competing technologies was made using the PCA procedure summarized in "Thickness Design for Concrete" [28] and the RCC fatigue properties given in "Structural Design of Roller Compacted Concrete for Industrial Pavements" [15]. The data used for the comparison is identical to Design 3-a given in Reference [28]. Tables E.1 through E.4 show the structural design calculations for both conventional and RCC pavements. It is clear from the tables that the RCC pavement must be thicker than the conventional one assuming the same traffic loading and level.

Table E.1/Structural Design Calculations (Conventional) (28)

CALCULATION OF CONCRETE PAVEMENT THICKNESS
(Use with Case I Single & Tandem Axle Design Charts)

Project DESIGN THREE-A
Type Rural Primary - Rolling Terrain No. of Lanes 2
Subgrade k 100 pci., Subbase 4-in. Granular Untreated
Combined k 130 pci., Load Safety Factor 1.1 (L.S.F.)

PROCEDURE

1. Fill in Col. 1, 2 and 6, listing axle loads in decreasing order.
2. Assume 1st trial depth. Use 1/2-in. increments.
3. Analyze 1st trial depth by completing columns 3, 4, 5 and 7.
4. Analyze other trial depths, varying M.R.*, slab depth and subbase type.**

1	2	3	4	5	6	7
Axle Loads kips	Axle Loads X L/L.S.F. kips	Stress psi	Stress Ratios	Allowable Repetitions No.	Expected Repetitions No.	Fatigue Resistance Used*** percent
Trial depth 8.0 in. M.R.* 650 psi k 130 pci						
SINGLE AXLES						
30	33.0	390	.58	57,000	870	2
28	30.8	360	.55	130,000	870	1
26	28.6	338	.52	300,000	1750	1
24	26.4	318	4.50	Unlimited	46,000	0
22	24.2		"	"	180,320	0
20	22.0		"	"	504,620	
TANDEM AXLES						
54	59.4	412	.63	14,000	870	6
52	57.2	402	.62	18,000	870	5
50	55.0	386	.59	42,000	8560	20
48	52.8	372	.57	75,000	8560	11
46	50.6	360	.55	130,000	13,570	10
44	48.4	348	.54	180,000	42,440	24
42	46.2	333	.51	400,000	48,300	12
40	44.0	318	4.50	Unlimited	69,920	0
					Total =	92

* M.R. Modulus of Rupture for 3rd pt. loading.

** Cement-treated subbases result in greatly increased combined k values.

*** Total fatigue resistance used should not exceed about 125 percent.

Table E.2/ Structural Design Calculations I (RCC)

CALCULATION OF CONCRETE PAVEMENT THICKNESS

Subgrade k 100 pci., Subbase 4-in. Granular Untreated

Combined k 130 pci., Load Safety Factor 1.1 (L.S.F.)

1	2	3	4	5	6	7
Axle Loads kips	Axle Loads X*L.S.F. kips	Stress psi	Stress Ratios	Allowable Repetitions No.	Expected Repetitions No.	Fatigue Resistance Used percent

Trial depth 8.75 in. M.R. 685 psi k 130 pci

SINGLE AXLES

30	33.0	330	0.48	76000	870	1
28	30.8	315	0.46	130000	870	1
26	28.6	305	0.45	165000	1750	1
24	26.4	290	0.42	360000	46000	13
22	24.2	255	0.37	∞	180320	0
20	22.0			∞	504620	0

TANDEM AXLES

54	59.4	365	0.53	21000	870	4
51	57.2	350	0.51	35000	870	3
50	55.0	340	0.50	46000	8560	19
48	52.8	330	0.48	76000	8560	11
46	50.6	320	0.47	100000	13570	14
44	48.4	310	0.45	165000	42410	26
42	46.1	295	0.43	280000	48300	17
40	44.0	280	0.41	465000	69920	15

Total = 125

Table E.3/ Structural Design Calculations II (RCC)

CALCULATION OF CONCRETE PAVEMENT THICKNESS

Subgrade k 100 pci., Subbase 4-in. Granular Untreated

Combined k 130 pci., Load Safety Factor 1.1 (L.S.F.)

1	2	3	4	5	6	7
Axle Loads kips	Axle Loads X*L.S.F. kips	Stress psi	Stress Ratios	Allowable Repetitions No.	Expected Repetitions No.	Fatigue Resistance Used percent

Trial depth 8.0 in. M.R. 650 psi k 130 pci

SINGLE AXLES

30	33.0	380	0.58	5800	870	15
28	30.8	360	0.55	12000	870	7
26	28.6	338	0.52	27000	1750	6
24	26.4	318	0.49	59000	46000	78
22	24.2	290	0.45	165000	180320	109
20	22.0	270	0.42	360000	504620	140

TANDEM AXLES

54	59.4	412	0.63	1600	870	54
51	57.2	402	0.62	2100	870	41
50	55.0	386	0.59	4500	8580	190
48	52.8	372	0.57	7500	8560	114
46	50.6	360	0.55	12000	13570	113
44	48.4	348	0.54	16000	42410	265
42	46.1	333	0.51	35000	48300	138
40	44.0	318	0.49	59000	69920	119

Total = 1389

Table E.4/ Structural Design Calculations III (RCC)

CALCULATION OF CONCRETE PAVEMENT THICKNESS

Subgrade k 100 pci., Subbase 4-in. Granular Untreated

Combined k 130 pci., Load Safety Factor 1.1 (L.S.F.)

1	2	3	4	5	6	7
Axle Loads kips	Axle Loads X*L.S.F. kips	Stress psi	Stress Ratios	Allowable Repetitions No.	Expected Repetitions No.	Fatigue Resistance Used percent

Trial depth 9.25 in. M.R. 650 psi k 130 pci

SINGLE AXLES

30	33.0	310	0.48	76 000	870	1
28	30.8	295	0.45	165 000	870	0
26	28.6	275	0.42	360 000	1750	0
24	26.4	265	0.41	465 000	46 000	10
22	24.2			∞	180320	0
20	22.0			∞	504620	0

TANDEM AXLES

54	59.4	340	0.52	27000	870	3
51	57.2	325	0.50	46000	870	2
50	55.0	315	0.48	76000	8560	11
48	52.8	305	0.46	130000	8560	7
46	50.6	295	0.45	165000	13570	8
44	48.4	285	0.44	210 000	42410	20
42	46.1	275	0.42	360000	18300	13
40	44.0	260	0.4	∞	69920	0

Total = 75